

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CPR OVERHEAD AT SELIM HILL
HIGHWAY 17
DISTRICT OF THUNDER BAY (UNORGANIZED), ONTARIO**

G.W.P 6108-10-01, Site No. 48C-25

Geocres Number: 42D-35

Report to

MMM Group Limited

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed replacement of the existing overhead structure which carries Highway 17 over Canadian Pacific (CP) rail line, M10.34 of the Nipigon Subdivision. The site is located approximately 6.5 km east of Rossport and approximately 13.7 km west of Schreiber, in the District of Thunder Bay (unorganized), Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

2 SITE DESCRIPTION

The CPR Overhead structure is located approximately 6.5 km east of Rossport and approximately 13.7 km west of Schreiber, in the District of Thunder Bay (unorganized), Ontario. At this location, Highway 17 travels in a predominantly northwest to southeast direction along the north edge of Lake Superior. For the purpose of this report, reference will be made to the existing west and east abutments.

At present, Highway 17 crosses the CP rail line on a three-span structure supported on two piers and two abutments. The total length of the bridge is 30.4 m and width of 14.3 m. The bridge is oriented at a skew of approximately 39.5° to the rail line.

The railway at this location was constructed in an approximate 1.5 to 4.0 m deep rock cut. Fractured bedrock is exposed along the alignment of the existing east pier and partially exposed behind the alignment of the west pier.

Each pier is supported on three rectangular columns and the tops of the pier foundations are exposed. The columns at the east pier are supported on individual spread footings constructed either above the rock cut face (north and centre columns) or along the alignment of the rock cut (south column). The west pier columns are supported on one continuous strip footing constructed in front of the rock cut face.

The existing approach embankments behind the abutments are up to about 6.4 m in height. No evidence of instability was observed on the embankment side slopes.

Overhead hydro wires run along the west side of the railway and pass under the west span of the overhead structure. A wooden hydro pole is situated under the bridge near the centre of the west span.

The area surrounding the bridge is gently rolling and is generally covered with a mix of deciduous and coniferous trees. Outcropping bedrock was noted at and in the general vicinity of the CPR Overhead structure.

Photographs in Appendix C show the existing CPR Overhead structure at Selim Hill and the general nature of the site.

The site lies within the physiographic region known as the Wawa Subprovince of the Superior Province of the Canadian Shield. Based on Ontario Geological Survey (OGS) Map 2518, titled “Surficial Geology of Northern Ontario”, dated 1987, the site is located in an area of “bare bedrock with thin glacial sediment cover”. Based on OGS Map 2545, titled “Bedrock Geology of Ontario”, dated 1991, the bedrock is of the Archean age and consists of intrusive rocks, mainly massive to foliated granodiorite and granite.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out in two stages. Between November 12 and 14, 2011, a total of four boreholes numbered SEL-01 to SEL-04 were advanced from the top of Highway 17. On each side of the existing overhead structure, one borehole was located in proximity to the abutment and one borehole within the approach embankment areas. Between April 23 and 27, 2014, an additional five boreholes numbered SEL-05 to SEL-09 were drilled beneath the deck structure on both sides of the CPR tracks.

The approximate borehole locations are shown on the enclosed Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

During the first stage of the investigation, Boreholes SEL-01 to SEL-04 were drilled using a truck mounted drill rig with hollow stem augers, NW casing and wash boring techniques. Boreholes SEL-01

and 04 were drilled within the existing west and east approaches to the structure, respectively, and were terminated at depths of 3.4 and 5.9 m (elevations 214.4 and 215.9 m), where refusal to auger penetration was encountered. Boreholes SEL-02 and 03 were drilled near the existing abutments to depths of 9.8 m and 7.8 m (elevations 209.1 to 213.3 m). Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Bedrock was proved in boreholes SEL-02 and SEL-03 by diamond coring in NQ size. The boreholes were advanced 3.4 and 2.9 m into bedrock.

Dynamic Cone Penetration Tests (DCPT) were conducted adjacent to borehole SEL-02 and 03 to supplement the subsurface information. The DCPT's were terminated at 5.7 and 6.0 m depths (elevation 213.2 to 215.1 m), upon refusal on probable bedrock.

During the second stage of the investigation, Boreholes SEL-05 to SEL-09 were advanced to depths ranging from 4.6 m to 7.9 m (between Elev. 208.3 m and 207.0 m) using portable equipment to core bedrock in BTQ size.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's field staff. The boreholes were logged in the field and recovered soil and rock samples were processed and transported to Thurber's geotechnical testing laboratory in Oakville, Ontario for further examination and testing.

All rock cores were logged and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions were observed in the open boreholes during and upon completion of drilling operations. Standpipe piezometers were installed in two boreholes to measure groundwater levels. The piezometers consisted of 19 mm PVC pipes with slotted screen and were enclosed in filter sand. The piezometers were subsequently decommissioned in general accordance with MOE Regulation 903 following completion of the final water level reading. The piezometer installation and borehole completion details are summarized in Table 3.1.

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The borehole co-ordinates and the ground surface elevations were obtained from the drawings provided by MMM Group Limited.

Table 3.1 - Borehole Completion Data and Piezometer installation Details

Borehole Number	Borehole Termination Depth/Elevation (m)	Piezometer Tip Depth/ Elevation (m)	Backfill Details
SEL-01	3.4 / 214.4	None installed	Concrete to 0.6 m, cuttings to 0.3 m, then asphalt to surface.
SEL-02	9.8 / 209.1	7.7 / 211.2	Sand to 4.1 m, bentonite holeplug to 1.9 m, concrete to 0.8 m, sand to 0.2 m, then asphalt to surface
SEL-03	7.8 / 213.3	4.9 / 216.2	Sand to 2.7 m, bentonite holeplug to 0.9 m, concrete to 0.2 m, then asphalt to surface.
SEL-04	5.9 / 215.9	None installed	Bentonite holeplug to 1.8 m, cuttings to 1.0 m, concrete to 0.15 m, then asphalt to surface.
SEL-05	4.7 / 208.3	None installed	Bentonite holeplug to 0.1 m then sand to surface.
SEL-06	4.6 / 207.9	None installed	Bentonite holeplug to 0.1 m then sand to surface.
SEL-07	4.6 / 207.4	None installed	Bentonite holeplug to 0.3 m then sand to surface.
SEL-08	7.9 / 207.2	None installed	Bentonite holeplug to 1.0 m then sand to surface.
SEL-09	7.7 / 207.0	None installed	Bentonite holeplug to 0.6 m then sand to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Grain size distribution tests (hydrometer and sieve) were carried out on selected samples. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the unconfined compressive strength of the bedrock. Results of point load tests on the rock core samples are enclosed in Appendix B, and are also presented on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the Borehole Locations and Soil Strata drawing in Appendix F. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

In general, the stratigraphy encountered at the overhead site consists of pavement structure overlying granular embankment fill, which in turn overlies the granite bedrock. In the boreholes drilled from the top of the highway embankment, bedrock and possible bedrock (interpreted as refusal to auger penetration) were encountered below the embankment fill at depths ranging from 3.4 to 6.4 m (Elev. 216.2 to 212.5 m). In the boreholes drilled under the structure on both sides of the CPR tracks, bedrock was either outcropping or encountered beneath shallow fill between Elev. 212 m and 214.7 m.

More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

A pavement structure, consisting of 125 mm of asphalt, was encountered in all of the boreholes drilled from the top of the highway embankment, i.e., in Boreholes SEL-01 to SEL-04.

5.2 Gravelly Sand to Sand and Gravel Fill

Cohesionless embankment fill consisting of gravelly sand to sand and gravel with trace fines (silt and clay) was encountered directly below the pavement structure in Boreholes SEL-01 to SEL-04. Where fully penetrated in Boreholes SEL-02 and SEL-03, the underside of this layer was at 6.4 and 4.9 m depth below the ground surface, or at Elev. 212.5 and 216.2, respectively. Boreholes SEL-01 and SEL-04 were terminated in the fill upon refusal to auger penetration at 3.4 m and 5.9 m depth (Elev. 214.4 and 215.9).

SPT N-values recorded in the granular fill ranged from 5 to 40 blows for 0.3 m penetration, indicating a loose to dense relative consistency. SPT N-values of 50 blows per 0.075 and 0.125 m of penetration were recorded in proximity to the bedrock surface in Borehole SEL-01, and near the ground surface in Borehole SEL-04, where cobbles may be present in the fill.

A fill consisting of sand with some gravel was encountered at the ground surface in Boreholes SEL-05 and SEL-08, drilled underneath of the structure. The fill was 0.2 m and 0.5 m thick and extended to Elev. 212.8 m and 214.6 m in Boreholes SEL-05 and SEL-08, respectively.

The moisture content of samples of the fill ranged from 1% to 5%, except for a single moisture content of 13% measured near the base of the fill in Borehole SEL-03.

Three samples of fill were selected for laboratory grain size analysis testing. The results of the tests are summarized below and are presented on the corresponding Record of Borehole sheets included in Appendix A. The grain size distribution curves for the samples are plotted on Figures B1 and B2 in Appendix B.

Soil Particles	Percentage %
Gravel	30 to 37
Sand	58 to 65
Silt and Clay	3 to 7

5.3 Bedrock

Granite and granodiorite bedrock was encountered and proved by coring in Boreholes SEL-02, SEL-03 and SEL-05 to SEL-09. Boreholes SEL-01 and 04 were terminated upon auger refusal on probable bedrock or on boulders. Details of rock coring are presented on the Record of Borehole sheets in Appendix A.

The bedrock is generally described as foliated, moderately weathered to fresh, grey to reddish brown granite and granodiorite with occasional light grey quartz inclusions.

Table 5.1, below, summarizes the depth to bedrock and bedrock surface elevations determined at the borehole locations.

Table 5.1 - Depths and Elevations of Bedrock Surface or Auger Refusal

Borehole Location/ Nearest Existing Foundation Units	Borehole Number	Top of Bedrock or Auger Refusal	
		Depth (m)	Elevation (m)
East Approach	SEL-04 ^(2, 3)	5.9	215.9
East Abutment	SEL-03 ^(1, 3)	4.9	216.2
	DCPT	6.0	215.1
East side of tracks	SEL-08 ⁽¹⁾	0.5	214.6
	SEL-09 ⁽¹⁾	0.0	214.7
West side of tracks	SEL-05 ⁽¹⁾	0.2	212.8
	SEL-06 ⁽¹⁾	0.0	212.5
	SEL-07 ⁽¹⁾	0.0	212.0
West Abutment	SEL-02 ^(1, 3)	6.4	212.5
	DCPT	5.7	213.2
West Approach	SEL-01 ^(2,3)	3.4	214.4

⁽¹⁾Bedrock proved by coring

⁽²⁾Auger refusal

⁽³⁾Borehole drilled from the top of the approach embankment

Dynamic cone penetration tests (DCPT) were conducted at a distance of 1 m and 0.5 m from Boreholes SEL-02 and SEL-03. The refusal to cone penetration were encountered at depth of 5.7 m (Elev.213.2) near Borehole SEL-02, and at 6.0 m (Elev.215.1) near Borehole SEL-03. These depths are approximately 0.7 m above and 1.1 m below the bedrock surface proved in the nearby Boreholes SEL-02 and SEL-03, respectively. The variations in the bedrock surface even within the short distance from the borehole locations should be anticipated.

Total Core Recovery (TCR) in the bedrock ranged from 94% to 100%. The Rock Quality Designation (RQD) determined from the recovered cores ranged from 0 to 100%, indicating a very poor to excellent rock quality. The upper 0.7 m to 3.5 m of bedrock was of a very poor to poor quality with frequent fractured zones and foliations. A fair to excellent quality bedrock was typically encountered below Elev. 209.5 on the west side of the tracks and below

Elev. 211.5 on the east side of the tracks. In Borehole SEL-03, drilled approximately 3 m east of the east abutment, a fair quality bedrock was encountered as high as at Elev. 215.5, approximately 0.7 m below the bedrock surface. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 17.

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests conducted on the rock core samples, ranges from 35.9 MPa to in excess of 250 MPa, indicating a medium strong to extremely strong intact rock. One value of UCS of 20.3 MPa was obtained for a rock sample retrieved from 2.7 m depth in Borehole SEL-08; this value indicates a weaker zone in the rock mass.

The point load test results (average values) carried out for rock samples are included on the Record of Borehole sheets in Appendix A, and the Point Load Test results are enclosed in Appendix B.

5.4 Water Levels

The water levels in the boreholes were observed during drilling and upon completion of drilling in overburden. Water was used during the coring operations and therefore the measured water levels may not reflect prevailing groundwater levels at the site. Two standpipe piezometers were installed in Boreholes SEL-02 and SEL-03 to monitor water levels after completion of drilling. The water levels measured in the piezometer and observed in open boreholes advanced in overburden are summarized in Table 5.2.

Table 5.2 - Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Comments
			Depth	Elevation	
West Abutment	SEL-01	Nov. 14, 2011	Dry	-	Open borehole
	SEL-02	Nov. 14, 2011	Dry*		In piezometer
		Nov. 30, 2011	7.7	211.2	
		May 29, 2012	7.2	211.7	
East Abutment	SEL-03	Nov. 14, 2011 Nov. 30, 2011	Dry* -	-	Piezometer Damaged
	SEL-04	Nov. 12, 2011	Dry	-	Open borehole

*Borehole dry during drilling in overburden.

Piezometric readings indicate that the water level is below the bedrock surface.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after intensive or prolonged periods of precipitation.

6 MISCELLANEOUS

Eastern Ontario Diamond Drilling Ltd. from Ottawa supplied the truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations. To advance boreholes below the existing overhead structure, portable coring equipment (Hilti System) was used.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger, C.E.T. of Thurber Engineering Ltd. Routine laboratory testing was carried out by Thurber Engineering Ltd.

Mr. Mark Farrant, P.Eng. directed the field operations.

The report was prepared by Ms. Anna Piascik, P.Eng., and reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects.

THURBER ENGINEERING LTD.

Anna Piascik, P.Eng.
Senior Geotechnical Engineer



Murray R. Anderson, P.Eng
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement bridge.

At present, Highway 17 crosses the CP rail line on a three-span structure constructed in 1952 and rehabilitated in 1999. A total length of the structure, including concrete wing walls is 42.2 m (30.4 m between abutments) and is 14.3 m in width. The deck is supported on two piers and two abutments founded on spread footings placed on bedrock. The bridge is oriented at a skew of approximately 39.5° to the rail line. The existing approach embankments are as much as 6.4 m in height at the overhead structure.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, the proposed replacement overhead will be a single-span structure 21.0 m in length and 14.6 m in width. The structure will be founded on 914 mm diameter reinforced concrete columns socketed into bedrock. Crash walls will be installed between the columns at each abutment and will extend 4 m above the top of rails. Precast pre-stressed concrete box girders will then be spanning the abutment pile caps to support the deck. The overhead structure will be replaced on the same alignment as the existing structure. RSS walls will be constructed to contain the approach embankment fill.

The discussion and recommendations presented in this report are based on the information provided by MMM and on the factual data obtained in the course of the investigations.

8 STRUCTURE FOUNDATIONS

The soil stratigraphy encountered at this site consists of pavement structure over embankment fill, which in turn overlies bedrock. The fill consists of loose to very dense sand and gravel to gravelly sand with occasional cobbles. The embankment fill was encountered to as much as 6.4 m depth in the

boreholes. Granite and granodiorite bedrock, as well as auger refusal on probable bedrock or cobbles/boulders was encountered below the fill or extending from the ground surface. The bedrock was contacted and proved in seven boreholes to be between Elev. 212.0 and 216.2. The depths and elevations of the bedrock at the proposed east and west abutments proved by coring are summarized in Table 8.1.

Table 8.1 - Depths and Elevations of Top of Bedrock

Proposed Foundation Element	Borehole Number	Top of Bedrock or Auger Refusal	
		Depth (m)	Elevation (m)
East Abutment	SEL-08	0.5	214.6
	SEL-09	0.0	214.7
West Abutment	SEL-05	0.2	212.8
	SEL-06	0.0	212.5
	SEL-07	0.0	212.0

Piezometric readings indicate that the water level is below the bedrock surface.

Based on existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on bedrock
- Drilled Caissons (shafts)

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix D. In view of the exposed/shallow bedrock at the site, steel piles are not a suitable foundation option and are not addressed further.

8.1 Spread Footings on Bedrock

Spread footings founded on bedrock are feasible for support of the structure.

The recommended highest founding elevations and geotechnical resistances for spread footings placed on granite/granodiorite bedrock are provided in the table below.

Table 8.2 - Recommended Highest Founding Levels for Spread Footings

Foundation Element	Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)
West Abutment	209.5	3,000
East Abutment	211.2	3,000

The founding levels indicated in the table are recommended to place the footings on relatively sound bedrock below the level of the highly fractured rock material disturbed during construction of the railway cut and existing bridge foundations, as assessed by the rock quality

encountered in the recovered rock cores. Excavation of fractured bedrock will be required to achieve the recommended founding levels.

The geotechnical resistance at SLS is not expected to govern design of spread footings on granite/granodiorite bedrock. For working stress design (AREMA code), an allowable bearing capacity of 2,000 kPa is recommended.

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC 2006 Clause 6.7.3 and Clause 6.7.4.

8.1.1 Resistance to Lateral Forces

Resistance to lateral forces / sliding resistance between the footing concrete and the bedrock surface should be evaluated in accordance with the CHBDC 2006 using an ultimate friction factor of 0.7 for concrete poured on clean sound bedrock. A suitable factor should be applied to this value.

If the frictional component is insufficient to resist lateral forces, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock or grout is exceeded. For the horizontal resistance analysis of a dowel, a lower bound value of 10 MPa may be used for the ultimate compressive strength of the rock. This design value includes a reduction factor to account for the rock fracturing. Dowels should be embedded at least 1.0 m into the rock.

8.1.2 Foundation Subgrade Preparation

The bases of the foundation excavations must be inspected by a geotechnical engineer to confirm that the exposed founding surfaces conform to the design requirements, and have been adequately prepared to receive concrete. Any loose or shattered rock must be removed and the footing founded on competent/sound bedrock.

Where sub-excavation is required to remove loose or shattered rock and other unsuitable material from below the design founding level, the founding surface should be re-established using concrete fill of the same class as that used for the footing.

8.2 Drilled Caissons

Drilled caissons socketed into bedrock may be employed to support the structural loads at this site. The preliminary GA drawing indicates that the overhead structure loads at each abutment will be supported on four columns 914 mm diameter each founded on drilled caissons socketed into bedrock. The preliminary axial load demand provided by the structural designer is 1430 kN per caisson. The horizontal load is about 70 kN at ULS.

8.2.1 Axial Resistance

Drilled caissons can be designed for axial geotechnical resistance provided by end bearing and shaft adhesion in the socket. To consider the end bearing geotechnical resistance, the base of the caisson will need to be cleaned and inspected. At this site where the intact rock is medium strong to extremely strong, the value of average shaft adhesion along the rock socket will be governed by the concrete compressive strength (minimum 30 MPa concrete assumed).

For design, the bedrock surface elevations at the abutments provided in Table 8.3 are recommended.

Table 8.3 - Bedrock Elevations Recommended for Design

	West Abutment	East Abutment
Elevation of Top of Fractured Bedrock	212.0	214.6
Elevation of Sound Bedrock	209.5	211.2

Based on the preliminary structural axial load demand of 1430 kN per caisson provided by the structural designer, several options for the design of the sockets could be considered. The axial geotechnical resistance estimated for several shaft diameters and depths below the design top of bedrock (Table 8.3) are presented in Table 8.4, below.

Table 8.4 - Recommended Resistance Values for Caisson Design

Socket Diameter (m)	Socket Length below Bedrock Surface/Base Elevation (m)		Factored Geotechnical Resistance at ULS (kN)
	West Abutment	East Abutment	
0.458	4.2 / 207.8	5.1 / 209.5	1450
	4.5 / 207.5	5.4 / 209.2	1730
	5.0 / 207.0	5.9 / 208.7	2100
0.610	3.8 / 208.2	4.7 / 210.0	1450
	4.0 / 208.0	4.9 / 209.7	1730
	4.5 / 207.7	5.4 / 209.2	2300
0.914	3.5 / 208.5	4.4 / 210.2	1700
	4.0 / 208.0	4.9 / 209.7	3000
1.2	3.2 / 208.8	4.1 / 210.5	1600
	4.0 / 208.0	4.9 / 209.7	3500

The SLS condition will not govern for caissons socketed into the rock. For working stress design (AREMA code), the resistance values presented in Table 8.4 may be taken as allowable capacities for design of the various caisson options.

The assessment of rock socket depth accounts for the presence of fractured rock extending beneath the bedrock surface to as much as 2.5 m at the west abutment and 3.4 m depth at the east abutment. A design based on shaft adhesion in the rock socket is recommended to avoid the need for cleaning and inspection of the base of a relatively small diameter shaft.

8.2.2 Lateral Resistance of Sockets

An ultimate lateral resistance of 2 MPa at factored ULS may be assumed for computation of the lateral capacity of the rock socket. The highly fractured rock above the track level cannot be relied upon to provide lateral support and should be neglected in the design.

8.2.3 Caisson Installation

Caisson installation must be in accordance with OPSS 903.

Along the new abutment alignments, bedrock was exposed at the ground surface or covered with a thin layer of fill. Highly fractured rock was encountered in the boreholes to depths of up to 2.5 m on the west side and 3.4 m on the east side of the tracks. The rock became relatively intact and medium strong to extremely strong below these depths.

When selecting equipment to excavate the rock socket, the high strength and variable quality of the bedrock will have to be considered. Excavation of the fill and portions of the highly fractured bedrock overlying the sound bedrock may be preferred to expedite drilling/coring operations in the sound bedrock. The installation method should ensure that the highly fractured material does not fall into the caisson.

The caissons will be installed in close proximity to the spread footings supporting the existing piers. It is recommended that construction of the caissons be carried out in a manner that does not disturb the existing footings supporting the remaining side of the bridge still carrying traffic during the initial stage of construction. Blasting to facilitate the rock removal is not permitted.

Selection of the methods and equipment employed for construction is the responsibility of the Contractor. The contract documents should contain a statement to alert bidders of the above construction concerns. Suggested wording for an NSSP addressing this issue is included in Appendix E.

8.3 Recommended Foundation

From a geotechnical perspective, spread footings founded on bedrock and drilled caissons socketed into the rock are considered to be feasible foundation options for the site conditions. Use of caisson foundations is preferred to minimize excavation of the fractured bedrock and potentially avoid the need for removal of the existing structure footings.

8.4 Frost Cover

The design depth of frost penetration at this site is 2.2 m. However, frost penetration is not an issue for footings bearing on bedrock or concrete fill placed on bedrock.

9 RETAINED SOIL SYSTEMS

The preliminary GA drawing indicates that Retained Soil System (RSS) walls are proposed for supporting embankment fill on both sides of the overhead structure. On the drawing, the walls are shown as being founded on bedrock at the face of the abutments and stepping upwards away from the abutment face, parallel to the roadway, to be founded within the embankment fill.

The attributes of the RSS walls used in conjunction with the new abutments should be defined as High Performance and High Appearance. The contract drawings and documents must include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile and cross-sectional space constraints. Special Provision for RSS walls SP599S22, December 2014 should be included.

The performance of a RSS is dependent, among other factors on the characteristics of its foundation.

The borehole information indicates that the front part of the RSS at each abutment face will be founded on bedrock. Preparation of the founding surface for construction of the RSS on bedrock should include exposing the bedrock surface by removal of the surficial fill and any loose rock debris, followed by placement of a granular engineered fill pad to achieve an even/flat surface on which to construct the RSS. The thickness of the granular pad will vary depending on the configuration of the bedrock surface.

Based on the borehole data and site observations, the elevation of the bedrock surface is expected to vary under the footprint of the RSS. Subject to the level of the exposed bedrock surface, adjustments to the wall design may be required during construction.

Further, the proximity of the RSS to the existing rock face may vary along the abutment alignment. Careful inspection of the exposed rock surface and the proximity of the proposed RSS to the rock face will be required during construction to determine the need for and develop remedial measures to deal with locations where the RSS nears the rock cut face. In general, the depressed zone (where present) between the rock cut face and the existing pier foundations should be backfilled to the RSS founding level (less 300 mm) using concrete fill.

If rock excavation is required, excavation must be carried out using pneumatic breakers or other methods that will avoid further shattering and disturbing the bedrock on which foundations will be constructed.

Sections of the RSS stepping upwards away from abutment will be constructed over the existing embankment fill. The embankment fill encountered in the boreholes generally consists of sand and gravel with occasional cobbles and is typically loose to compact. To improve the bearing resistance and reduce the potential for differential settlement along the sections of RSS founded on the existing embankment fill, placement of a minimum 1.0 m thick pad of granular material is recommended below the RSS base.

Granular fill placed below the RSS should consist of OPSS Granular “A” material compacted to 100% of its SPMDD at a moisture content within 2% of the optimum moisture content. The engineered fill pad must extend at least 1.0 m beyond the limits of the RSS mass and levelling strip.

RSS walls founded on bedrock or on granular fill prepared as noted above can be designed for a factored geotechnical resistance of 350 kPa at ULS and a reaction of 275 kPa at SLS.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall placed on engineered granular fill may be estimated using an ultimate friction coefficient of 0.55.

The global stability of the RSS wall placed on bedrock or granular pad is expected to be adequate.

10 APPROACH EMBANKMENTS

Based on the preliminary GA drawing dated November 2014, the proposed finished grade at the structure will be consistent with the existing grade. At the west abutment, the finished grade will be approximately at Elevation 219.3 m, and at the east abutment at Elev. 220.7. The existing embankments are inclined at approximately 2H:1V. No indication of embankment instability was reported or noted during the field investigation.

The stability of the existing embankments founded on bedrock is expected to be adequate.

11 BACKFILL TO ABUTMENTS

If consideration is given to the use of cast-in-place abutment walls, the backfill to the walls should consist of granular fill as specified in OPSD 3101.150.

Granular backfill to the abutments should consist of Granular A, Granular B Type II or Granular B Type III material meeting the requirements of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”. The backfill should conform to the specifications in OPSS 902, and be placed to the extents shown in OPSD 3101.150. All new embankment earth fill should be placed in regular lifts and be compacted in accordance with OPSS 501. Also, compaction equipment to be used adjacent to retaining structures must be restricted in accordance OPSS 501.

The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150.

12 LATERAL EARTH PRESSURES

Lateral earth pressures acting on the structure may be assumed to be triangularly distributed and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K (\gamma h + q)$$

Where:

p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 12.1.

Table 12.1 – Earth Pressure Coefficients (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I, Granular B Type III, or Existing Granular Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.47*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 12.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I and Type III or 1.7 m for Granular A or Granular B Type II.

13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.00
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.00
- Peak Ground Acceleration 0.011 g

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient S of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 13.1 may be used:

Table 13.1 – Earth Pressure Coefficients for Earthquake Loading

Loading Condition	Earth Pressure Coefficient (K) for Earthquake Loading			
	Granular A or Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ$; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.28	0.40	0.31	0.48
Passive (K_{PE})	3.7	-	3.2	-
At Rest (K_{OE})**	0.44	-	0.49	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods (1973).

Based on soil types and field test data, the foundation soils at the site are assessed as not being prone to liquefaction.

14 EXCAVATION AND GROUNDWATER CONTROL

Temporary excavation for footing construction, if selected for support of the structure, will generally extend through fractured bedrock to depths in the order of 2.5 to 3.4 m. Temporary excavation for RSS construction will extend through the existing embankment fill consisting of gravelly sand to sand and gravel. Based on the borehole data, the excavation depth of fill overlying the bedrock will vary from about 3.4 to 6.4 m.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the cohesionless fill within the probable depth of excavation at this site may be classed as Type 3 soils.

The excavation of the cohesionless soils and backfilling for foundations must be carried out in accordance with OPSS 902.

For construction staging, roadway protection must be supplied in accordance with OPSS 539 and designed for Performance Level 2. A soldier pile and lagging system is considered to be a suitable option for roadway protection, however socketing of the piles into bedrock will likely be required.

Discussions with the railway authorities should be undertaken to determine the required level of track protection if excavation for footing construction is planned. CP Rail may require a more stringent performance level for track protection (Performance Level 1).

Groundwater levels observed in the piezometer in Borehole SEL-02 indicate that seepage or perched water may be encountered within the fractured bedrock or in the fill immediately above the bedrock surface. It is anticipated that this water can be handled by proper surface drainage measures and localized sump pumping.

If rock excavation is required, excavation must be carried out using pneumatic breakers or other methods that will avoid shattering and disturbing the bedrock on which foundations will be constructed. Further, excavation must not disturb the existing footings supporting the remaining side of the bridge still carrying traffic during the initial stage of construction.

Fibre optic cables or other buried utilities might be present along the CP track in the vicinity of the new foundation areas. These utilities must not be undermined or damaged during construction of the foundation system for the new structure. The locations of any utilities should be established in relation to potential work zones, and if necessary exposed to protect them during construction of the new foundations. Relocation of, and/or special protective measures for affected utilities may be required.

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Excavation for caisson construction

Excavation for caisson construction in rock at this site must consider the presence of highly fractured rock in the upper zone of the bedrock, as well as moderately to extremely strong rock below the fractured zone. The Contractor must have the appropriate equipment to advance the holes to the design socket base without disturbing the bedrock forming the socket walls and base.

- Variable depth to bedrock surface during RSS construction

The bedrock surface elevation varies across the site, and when exposed during RSS construction, may vary between and away from the borehole locations. The thickness of the granular pad for the RSS base may have to be appropriately adjusted during construction. A note should be provided on the RSS drawing to state this possibility.

- Proximity of RSS to rock cut face

The proximity of the RSS to the existing rock cut face may vary along the abutment alignment. Careful inspection of the exposed rock surface and the proximity of the proposed RSS to the rock face will be required during construction to determine the need for and develop remedial measures to deal with locations where the RSS nears the rock cut face.

16 CLOSURE

Analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Murray Anderson, P.Eng. and Dr. P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Anna Piascik, P.Eng
Senior Geotechnical Engineer



Murray R. Anderson, P.Eng., M.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS


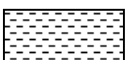

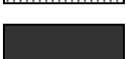

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
	(MPa)	(psi)	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No SEL-01

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 352.3 E 273 175.7 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.11.14 - 2011.11.14 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
							20	40	60	80	100						
217.8	GROUND SURFACE																
0.0	ASPHALT:(125mm)		1	GS													
0.1	SAND and GRAVEL trace silt, occasional cobbles Loose to Compact Brown Moist (FILL)		1	SS	29												
			2	SS	12												
			3	SS	9												
			4	SS	50/ 0.075												
214.4	END OF BOREHOLE AT 3.4m UPON AUGER REFUSAL. BOREHOLE OPEN TO 1.5m AND DRY. BOREHOLE BACKFILLED WITH CONCRETE TO 0.6m, CUTTINGS TO 0.3m THEN ASPHALT TO SURFACE.																
3.4																	

RECORD OF BOREHOLE No SEL-02

1 OF 2

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 351.7 E 273 193.9 ORIGINATED BY SLL
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.11.12 - 2011.11.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			FRACTURE INDEX /0.3m	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
218.9	GROUND SURFACE							20 40 60 80 100	20 40 60					GR SA SI CL
0.0	ASPHALT:(125mm)							20 40 60 80 100	20 40 60					
0.1	SAND, gravelly, trace silt, occasional cobbles Loose to Compact Brown Moist (FILL)		1	SS	18		218							DCPT located 1.0m N & 1.0m E of borehole
			2	SS	9		217							
			3	SS	10		216							
			4	SS	15		215							
			5	SS	8		214							
			6	SS	9		213							
212.5	Auger refusal at 6.4m and start coring													
6.4	GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown Sub-vertical fracture (125mm) at 6.4m Vertical fracture (250mm) at 6.7m Vertical joint from 7.6m to 8.4m		1	RUN			212						FI	RUN #1 TCR=100% SCR=45% RQD=45% UCS=115.7MPa (Average)
			2	RUN			211						5	RUN #2 TCR=100% SCR=0% RQD=0%
			3	RUN			210						4	RUN #3 TCR=100% SCR=5% RQD=0%
			4	RUN									3	
													2	RUN #4 TCR=100% SCR=72% RQD=69% UCS=112.8MPa (Average)
209.1	Vertical joint (350mm) at 9.4m												4	
9.8	END OF BOREHOLE AT 9.8m.													

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SEL-02

2 OF 2

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 351.7 E 273 193.9 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.11.12 - 2011.11.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
	Piezometer installation consists of 31mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Nov 14/11 Dry Nov 30/11 7.7 211.2 May 29/12 7.2 211.7																

RECORD OF BOREHOLE No SEL-03

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 373.2 E 273 220.4 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.11.13 - 2011.11.13 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
221.1	GROUND SURFACE							20 40 60 80 100	20 40 60			GR SA SI CL
0.0	ASPHALT:(125mm)						221	○ UNCONFINED + FIELD VANE	20 40 60			DCPT located 1.0m S & 0.5m W of borehole
0.1	SAND, gravelly, trace silt, occasional cobbles Compact to Dense Brown Moist (FILL)		1	SS	17		220	● QUICK TRIAXIAL × LAB VANE	20 40 60			
			2	SS	12		219					
			3	SS	40		218					
			4	SS	10		217					
			5	SS	10		216					
216.2	Start coring at 4.9m						215					
4.9	GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown		1	RUN			214					
	Vertical joint at: 200mm at 5.0m 150mm at 5.3m Rubble zone (50mm) at 5.2m		2	RUN								
	Sub-vertical joint at: 300mm at 5.6m 75mm at 6.9m		3	RUN								
213.3	END OF BOREHOLE AT 7.8m. Piezometer installation consists of 31mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.											
7.8	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Nov 14/11 Dry May 29/12 Blocked (Unable to read)											

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 2/19/15

+ 3, × 3: Numbers refer to
Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SEL-04

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 373.1 E 273 236.8 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.11.12 - 2011.11.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
221.9	GROUND SURFACE																
0.0	ASPHALT:(125mm)																
0.1	SAND, gravelly, trace silt, occasional cobbles. Loose to Very Dense Brown Moist (FILL)		1	SS	50/ 0.125												
			2	SS	16												
			3	SS	9												
			4	SS	19												
			5	SS	5												
			6	SS	13												
215.9	END OF BOREHOLE AT 5.9m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, CUTTINGS TO 1.0m, CONCRETE TO 0.15m THEN ASPHALT TO SURFACE.																
5.9																	

30 63 7
(SI+CL)

RECORD OF BOREHOLE No SEL-05

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 361.6 E 273 189.8 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE BTQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.04.26 - 2014.04.26 CHECKED BY MEF


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
213.0	GROUND SURFACE															GR SA SI CL	
0.0	SAND , trace gravel Frozen (FILL)																
0.2	GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown Rubble zone from 0.4m to 1.0m Sub-vertical joint (25mm thick) at 1.0m, 1.3m, 1.6m 200mm at 1.8m Rubble zone (125mm thick) at 1.7m Fractured zone (125mm) at 2.1m Sub-vertical joint (25mm thick) at 2.3m Vertical joint (250mm thick) at 2.3m Sub-vertical joint (50mm thick) at 3.1m, 3.4m and (100mm) at 3.0m Sub-vertical joint (25mm to 50mm thick) at 3.6m, 4.3m and (250mm) at 3.8m		1	RUN												RUN #1 TCR=100% SCR=31% RQD=0%	
			2	RUN			212										RUN #2 TCR=100% SCR=73% RQD=27% UCS=126.9MPa (Average)
			3	RUN			211										RUN #3 TCR=100% SCR=17% RQD=0%
			4	RUN			210										RUN #4 TCR=100% SCR=95% RQD=0% UCS=74.4MPa (Average)
			5	RUN			209										RUN #5 TCR=100% SCR=100% RQD=92% UCS=177.2MPa (Average)
208.3																	
4.7	END OF BOREHOLE AT 4.7m. BOREHOLE GROUTED WITH BENTONITE HOLEPLUG TO 0.1m, THEN SAND TO SURFACE.																

RECORD OF BOREHOLE No SEL-06

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 358.2 E 273 197.6 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE BTQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.04.24 - 2014.04.26 CHECKED BY MEF


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20 40 60 80 100										
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)				GR SA SI CL			
212.5	GROUND SURFACE																	
0.0	GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown Vertical joint (100mm thick) at surface Sub-vertical joint (25mm to 75mm thick) at 0.2m, 0.6m, 1.1m 225mm at 0.8m 300mm at 1.6m Rubble zone (75mm to 100mm thick) at 1.1m, 1.8m Sub-vertical joint (175mm thick) at 1.9m Rubble zone (75mm thick) at 2.1m Sub-vertical joint (25mm thick) at 3.3m		1	RUN														
			2	RUN														
			3	RUN														
			4	RUN														
			5	RUN														
			207.9															
4.6	END OF BOREHOLE AT 4.6m. BOREHOLE GROUTED WITH BENTONITE HOLEPLUG TO 0.1m, THEN SAND TO SURFACE.																	

RECORD OF BOREHOLE No SEL-07

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 355.0 E 273 204.9 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE BTQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.04.23 - 2014.04.23 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
212.0	GROUND SURFACE						20	40	60	80	100	20	40	60		GR SA SI CL		
0.0	GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown Sub-vertical joint at surface at: 100mm at 0.5m 150mm at 0.6m 125mm at 1.0m Quartz seam (200mm thick) at 1.5m Sub-vertical joint (75mm thick) at 1.7m, 2.1m Mechanical break at 2.5m Sub-horizontal joint (25mm thick) at 2.9m Mechanical break at 3.6m, 3.7m		1	RUN		211										RUN #1 TCR=100% SCR=54% RQD=0% UCS=287.9MPa (Average) RUN #2 TCR=94% SCR=94% RQD=33% UCS=243.8MPa (Average) RUN #3 TCR=100% SCR=100% RQD=85% UCS=74.7MPa (Average) RUN #4 TCR=100% SCR=100% RQD=90% UCS=148.6MPa (Average) RUN #5 TCR=100% SCR=100% RQD=100% UCS=200.3MPa (Average) RUN #6 TCR=100% SCR=100% RQD=91% UCS=241.5MPa (Average)		
			2	RUN														
			3	RUN				210										
			4	RUN			209											
			5	RUN														
			6	RUN				208										
207.4																		
4.6	END OF BOREHOLE AT 4.6m. BOREHOLE GROUTED WITH BENTONITE HOLEPLUG TO 0.3m, THEN SAND TO SURFACE.																	

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No SEL-09

1 OF 1

METRIC

WP# 6108-10-01 LOCATION Selim Hill/CPR N 5 411 365.7 E 273 220.9 ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE BTQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.04.26 - 2014.04.26 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	FRACTURE INDEX (0.3m)	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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214.7	GROUND SURFACE							20	40	60	80	100	20	40	60	GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
0.0	<p>GRANITE and GRANODIORITE with occasional light grey quartz inclusions, foliated, moderately weathered to fresh, grey to reddish brown</p> <p>Sub-vertical joint contain trace sandy material at: 75mm at 0.05m 150mm at 0.3m 225mm at 0.5m</p> <p>Highly fractured zone from 0.9m to 1.3m Sub-vertical joint through out the run</p> <p>Sub-vertical joint: 250mm at 1.9m 375mm at 2.3m Highly fractured zone from 2.8m to 3.4m</p> <p>Sub-vertical joint from 3.2m to 3.4m</p> <p>Horizontal joint at 3.6m, 3.8m</p> <p>Sub-vertical joint: 150mm at 3.8m 175mm at 4.1m Sub-vertical joint (50mm thick) at 4.4m, 4.9m 250mm at 4.6m</p> <p>Sub-vertical joint (25mm to 50mm thick) at 5.3m, 5.4m 150mm at 5.0m Horizontal joint at 5.2m</p> <p>Fractured zone 75mm at 5.5m</p> <p>Sub-horizontal joint (25mm to 50mm thick) at 5.6m, 5.8m, 5.9m 150mm at 6.3m Horizontal joint at 6.8m, 6.9m</p> <p>Subvertical joint (25mm to 75mm thick) at 7.1m, 7.3m, 7.5m</p>		1	RUN			214																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																

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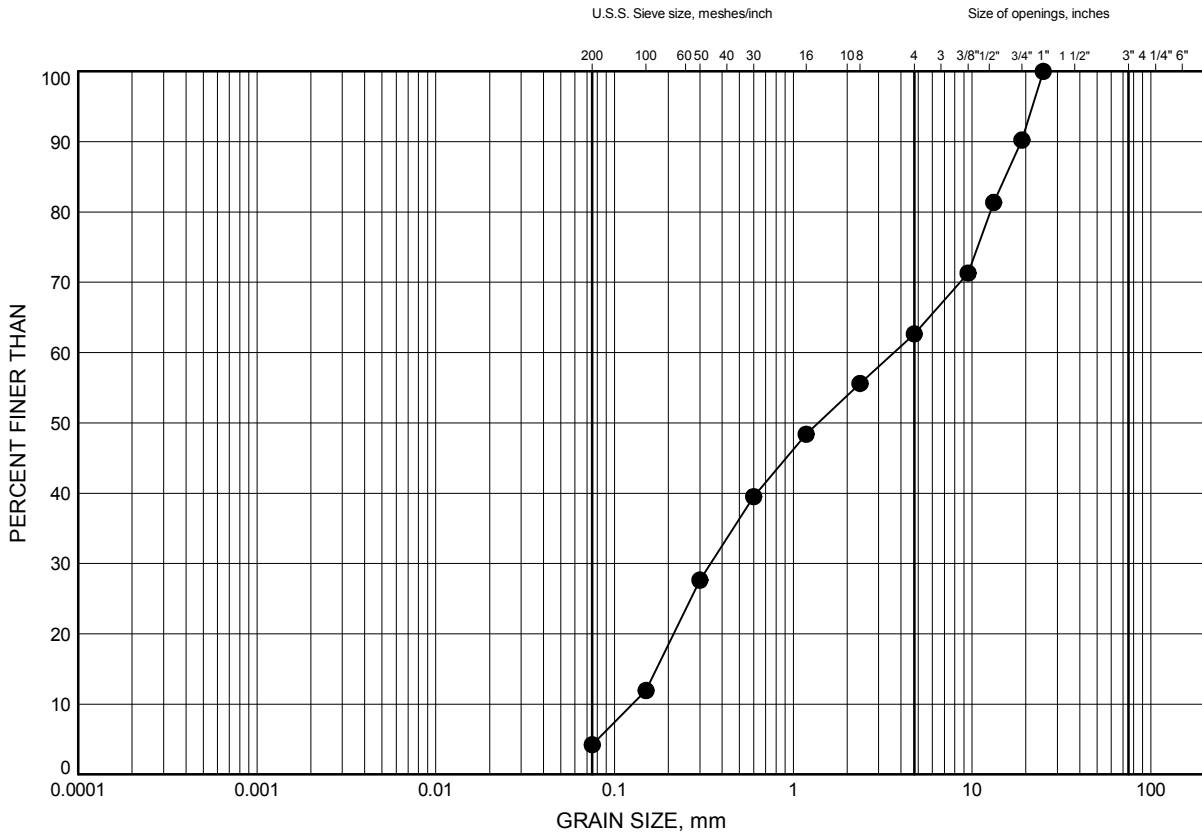
Appendix B

Laboratory Test Results

Selim Hill/CPR
GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SEL-01	1.83	215.99

Date December 2014
WP# 6108-10-01

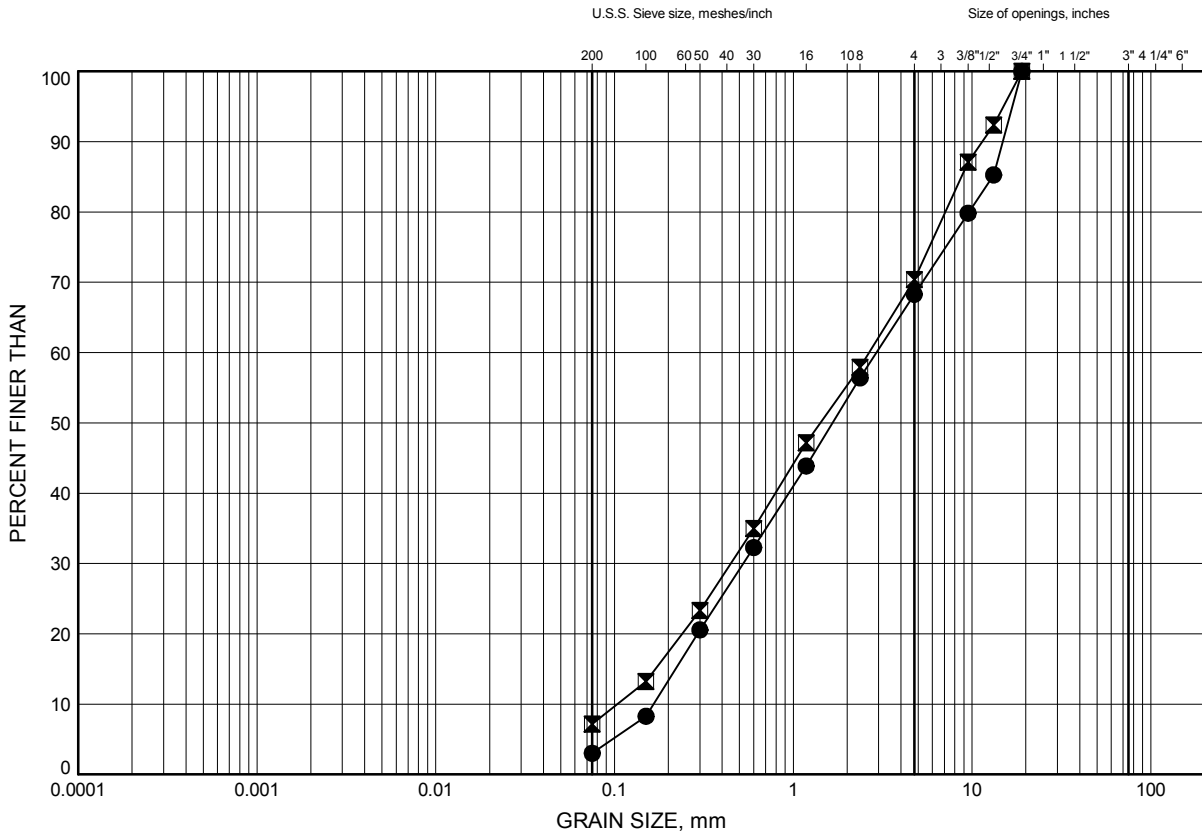


Prep'd AN
Chkd. AP

Selim Hill/CPR
GRAIN SIZE DISTRIBUTION

FIGURE B2

GRAVELLY SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SEL-02	2.59	216.28
◻	SEL-04	3.05	218.81

Date December 2014
WP# 6108-10-01



Prep'd AN
Chkd. AP

Appendix C

Site Photographs

Replacement of CPR Overhead at Selim Hill
Highway 17, Site 48C-25



Photograph 1 – Looking north at south side of overhead structure



Photograph 2 – Looking southwest on Highway 17 at overhead structure location



Photograph 3 – Existing east pier and abutment



Photograph 4 – North column of east pier



Photograph 5 – Centre column of east pier



Photograph 6 – South column of east pier



Photograph 7 – West pier and abutment



Photograph 8 – South column of west pier



Photograph 9 – Behind centre column of west pier



Photograph 10 – Behind north column of west pier



Photograph 11 – Hydro wires passing under west span of structure

Appendix D

Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Bedrock	Augered Caissons Socketed into Bedrock
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Removal of fractured rock to as much as 3.4 m depth will be required. ii. Possible disturbance to existing bridge foundations during excavation. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for caissons socketed into bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to spread footings. ii. Caisson construction will extend through highly fractured rock and into strong to extremely strong rock. iii. Potential difficulty in cleaning and inspecting rock sockets.
FEASIBLE	RECOMMENDED

Appendix E

List of Standard Specifications and Special Provisions

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 902
- OPSS 903
- OPSD 3101.150
- SP 110S13, Amended to OPSS 1010, April 2004
- SP 599S22

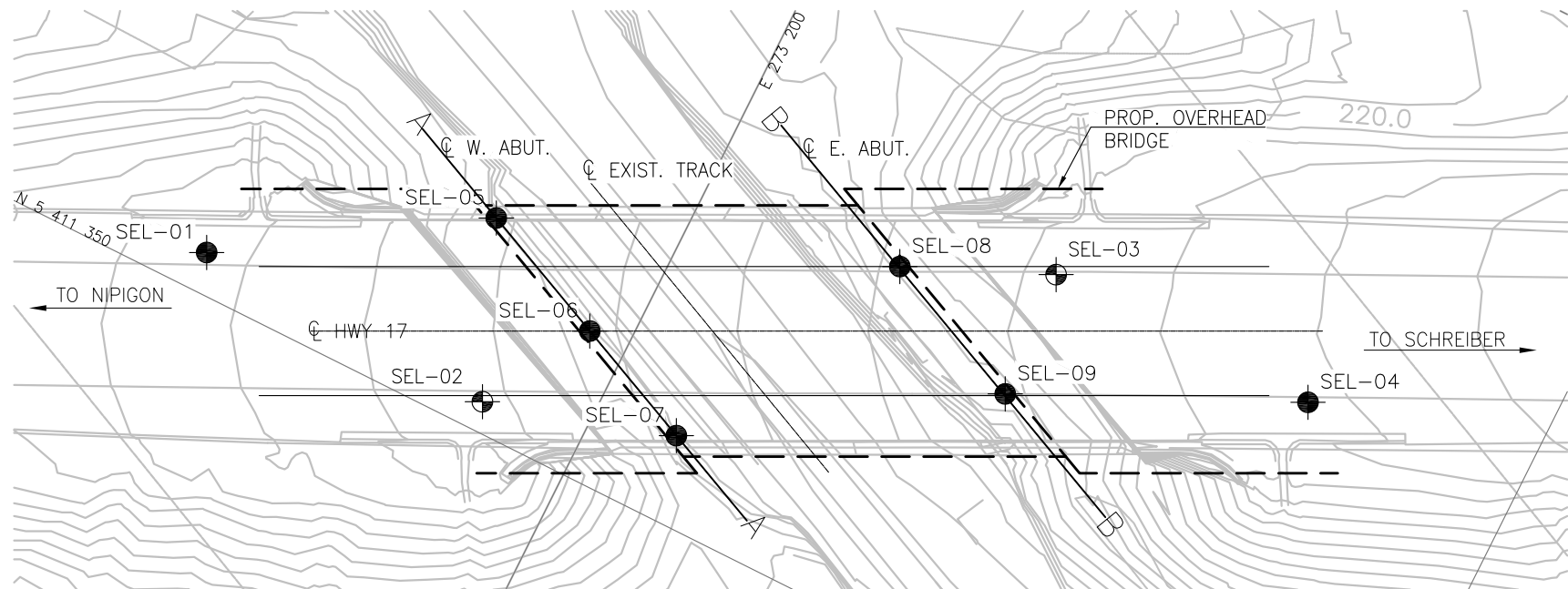
2. Suggested Text for NSSP on “Construction of Caissons”

Caisson installation shall be in accordance with OPSS 903. The Contractor is also advised of the following:

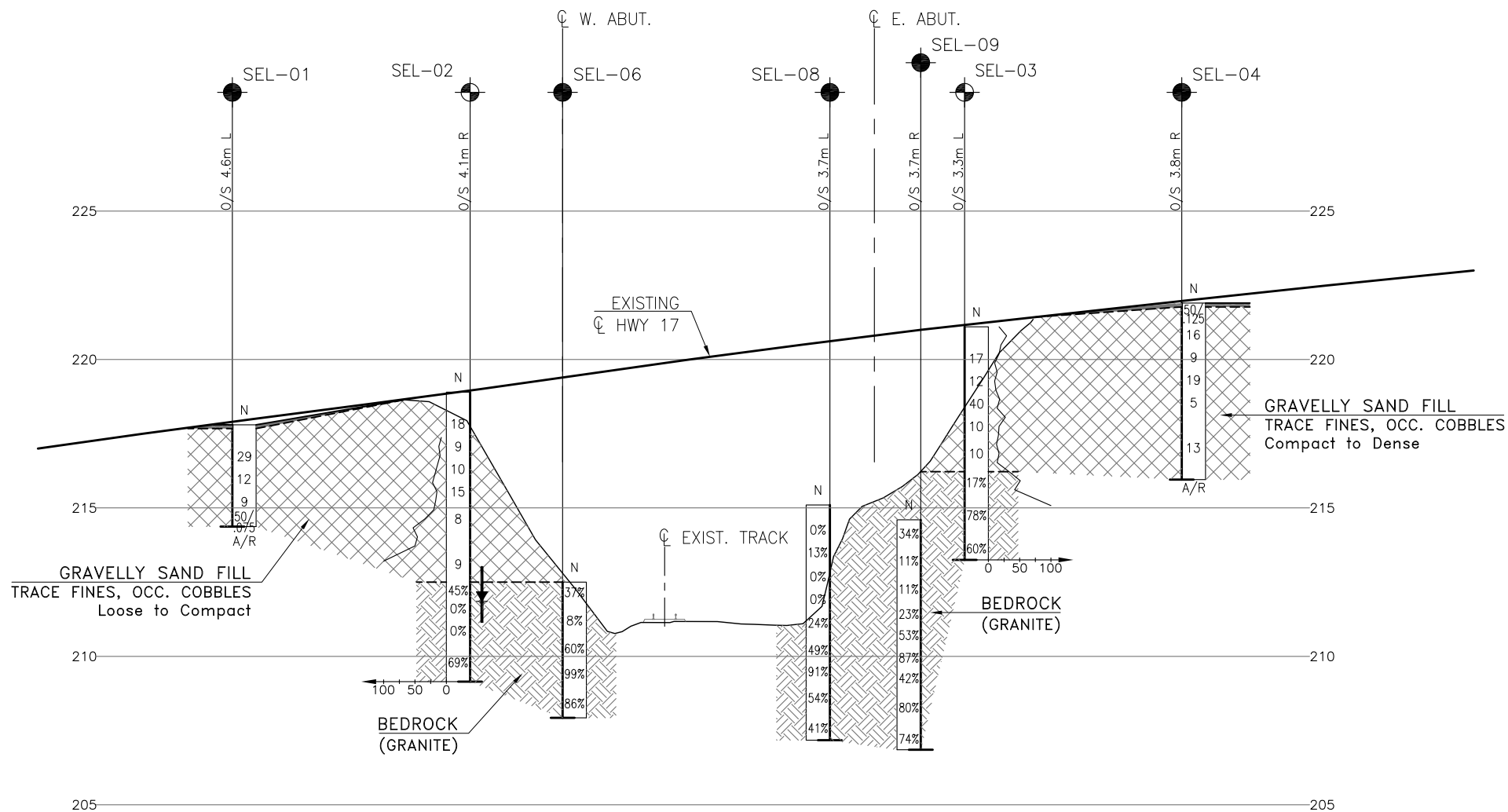
- Caisson installation will extend through surficial sand fill, highly fractured/disturbed bedrock, and into strong to extremely strong granite rock. The highly fractured nature of the upper zone of bedrock, the variable rock quality, and the high strength of the underlying sound bedrock must be taken into account when selecting equipment to advance the caisson into rock.
- Equipment supplied to construct the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The caissons will be installed in close proximity to the spread footings supporting the existing piers. Construction of the caissons must be carried out in a manner that does not disturb the existing footings supporting the remaining side of the bridge still carrying traffic during the initial stage of construction.
- The installation method should ensure that the highly fractured material does not fall into the caisson.

Appendix F

Borehole Locations and Soil Profile Drawing



PLAN
SCALE 1:400



PROFILE ALONG C HWY 17
H 1:400
V 1:200

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2015-6004
WP No 6108-10-01

HIGHWAY 17
SELIM HILL
CPR OVERHEAD
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
12



KEYPLAN

LEGEND

- Borehole
- Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level During Drilling
- Water Level in Piezometer
- Rock Quality Designation (RQD)
- Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SEL-01	217.8	5 411 352.3	273 175.7
SEL-02	218.9	5 411 351.7	273 193.9
SEL-03	221.1	5 411 373.2	273 220.4
SEL-04	221.9	5 411 373.1	273 236.8
SEL-05	213.0	5 411 361.6	273 189.8
SEL-06	212.5	5 411 358.2	273 197.6
SEL-07	212.0	5 411 355.0	273 204.9
SEL-08	215.1	5 411 369.6	273 212.1
SEL-09	214.7	5 411 365.7	273 220.9

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 42D-35

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AP	CHK	MA
DRAWN	AN	CHK	AP
CODE	LOAD	DATE	FEB 2015
SITE	48C-25	STRUCT	DWG 2

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2015-6004
WP No 6108-10-01

HIGHWAY 17
SELIM HILL
CPR OVERHEAD
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
13



KEYPLAN

LEGEND

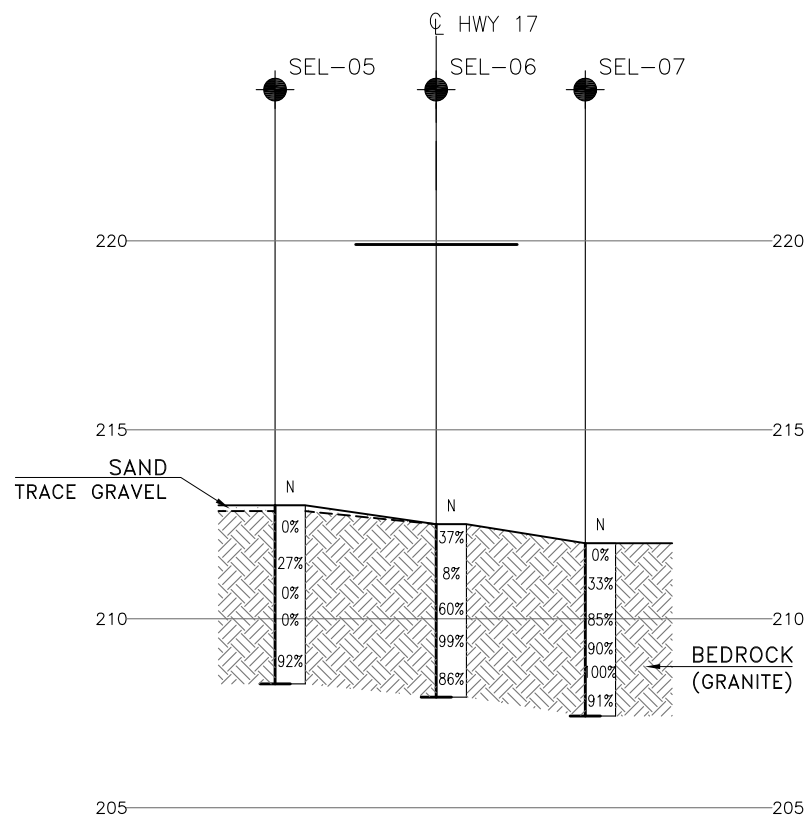
- Borehole
- Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level During Drilling
- Water Level in Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SEL-01	217.8	5 411 352.3	273 175.7
SEL-02	218.9	5 411 351.7	273 193.9
SEL-03	221.1	5 411 373.2	273 220.4
SEL-04	221.9	5 411 373.1	273 236.8
SEL-05	213.0	5 411 361.6	273 189.8
SEL-06	212.5	5 411 358.2	273 197.6
SEL-07	212.0	5 411 355.0	273 204.9
SEL-08	215.1	5 411 369.6	273 212.1
SEL-09	214.7	5 411 365.7	273 220.9

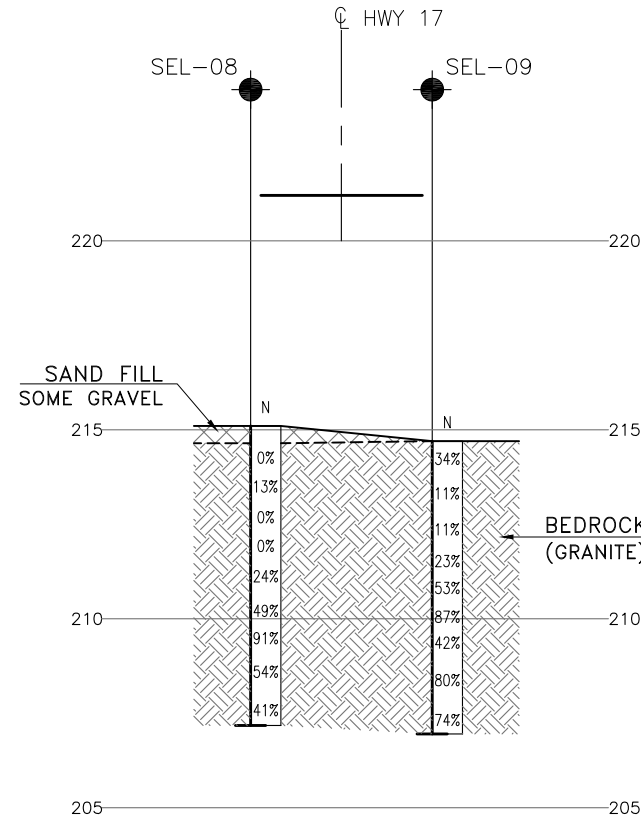
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 42D-35



SECTION ALONG A-A



SECTION ALONG B-B



H 1:400

V 1:200

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AP	CHK MA	CODE
DRAWN	AN	CHK AP	SITE 48C-25
			STRUCT
			DWG 3
			DATE FEB 2015